

CHAPTER 3

FISH PASSAGE DESIGN ELEMENTS

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3 FISH PASSAGE DESIGN ELEMENTS

This chapter describes design elements that are common to all fish passage road crossing types, regardless of the specific design method used to size the conveyance structure. It includes discussion of the following topics:

- data collection and site assessment needs
- hydrologic methods to determine fish passage flows
- basic culvert design for fish passage
- basic and applied hydraulic principles for fish passage road crossing design
- other considerations for the fish passage road crossing design process

In this chapter, emphasis is placed on how the process of designing road crossings for fish passage differs from the design process for traditional, non-fish passage road crossings. Details regarding specific design aspects of the alternative design methods are presented in the remaining chapters of this manual.

Items discussed in this chapter and elsewhere throughout the manual may refer to terms that are not commonly used in roadway engineering. Appendix A provides a glossary of terms used in fish passage road crossing design, defined with the usage common in California. These terms may vary slightly in their precise definition as compared to the usage in other states and countries.

3.1 Site Assessment for Fish Passage Road Crossing Design

The design of any road crossing installation requires the evaluation of a large amount of data. Guidelines for collection of this data can be found in many design guidelines for traditional, non-fish passage culverts, including the FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts", and HDM Chapter 800. This section discusses areas where fish passage analysis requires additional data or more precise data than normally used for culvert design.

These areas include:

- channel slope
- channel width
- channel cross sections
- channel roughness
- bed load composition
- channel stability

3.1.1 Channel Slope

Channel slope is a key factor in selecting the most appropriate fish passage road crossing type, and it plays a critical role in subsequent design development. Channels having slopes less than about 3 percent should be able to accommodate road crossings of any type. Channels with slopes in the moderate to high range of 3 percent to 6 percent are not likely to provide successful fish passage conditions for culverts designed through the active channel or hydraulic design methods. Bridges or streambed simulation culverts are the recommended road crossing types for sites having moderate to high channel slopes.

The channel slope should be determined using field data collected at the proposed road crossing

site. Data that defines the channel slope may already have been collected during the fish passage field assessment conducted by the Environmental Division. Therefore, check with the District Environmental Unit or the District Hydraulics Unit to obtain a copy of the completed Caltrans Fish Passage Data Collection forms developed for the site. If the forms have not been completed or they lack the channel slope information, use pages 4 and 5 of the Caltrans Fish Passage Data Collection - Second Pass Survey Information to collect the necessary information.

As compared to a survey done to determine channel slope for a non-fish passage road crossing, a survey for a fish passage road crossing requires significantly greater detail in the longitudinal profile along the deepest point (the thalweg) of the stream. Key features of interest for fish passage purposes are presented in Figure 3-1; background information regarding the relevance of these features to fish passage success can be found in Part IX of the California Salmonid Stream Habitat Restoration Manual (CDFG 2003).

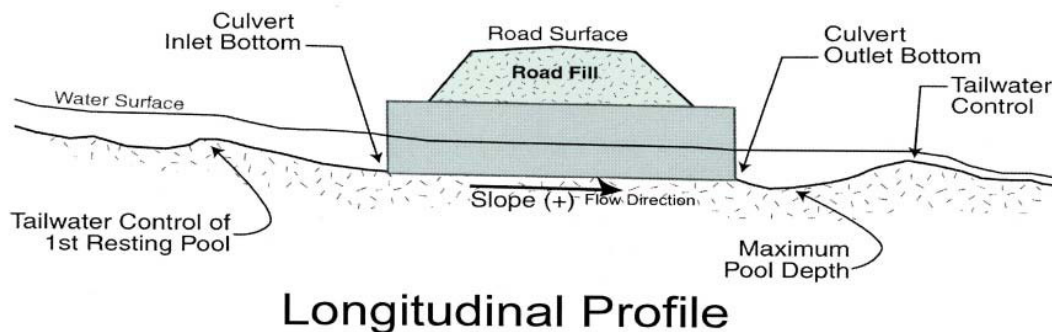


Figure 3-1. Key features of a longitudinal profile of interest for design of fish passage road crossings. (CDFG 2003)

HDS No. 5 indicates the channel slope for a traditional culvert design usually can be obtained using the lowest point from three channel cross-sections: one where the centerline of the proposed roadway intersects the centerline of the stream, and two more taken about 100 feet on either side of the crossing. In contrast, for a fish passage road crossing, it is much more important to determine the slope using data that includes the elevations of the tailwater control points above and below the crossing.

The tailwater control point below the culvert location is an especially important feature for fish passage design, because it is an elevation in the stream that may influence the depth of water in the culvert barrel. The tailwater control point is usually created by one of the following features (CDFG 2003):

- *Pool Tailout:* Commonly referred to as the riffle crest. Deposition of substrate downstream of the outlet pool controls the pool elevation.
- *Full-Spanning Log or Debris Jam:* Naturally deposited pieces of wood or trees that influence the outlet pool elevation.
- *Boulder, or Concrete Weirs:* These structures are often placed downstream of perched culverts to raise tailwater elevation and reduce the leap height required by migrating fish to enter a culvert.
- *No Control Point (Channel Cross-Section Recommended):* Describes situations where there is no outlet pool, allowing water to flow unimpeded downstream. In this situation the channel

roughness, slope, and cross-sectional shape govern the water elevation downstream of the outlet. When surveying a cross-section at these sites, it should be located within five feet of the outlet.

In cases where there is evidence of significant changes in channel slope in the vicinity of the culvert, it may be necessary to extend the limits of the longitudinal survey a significant distance upstream and downstream of the crossing location in order to determine an accurate assessment of the average slope conditions. This need may also apply to culvert replacement projects where the channel has been affected by the existing culvert and created a scour hole. With an elongated longitudinal survey, the designer can predict the natural channel slope and elevation at the crossing location by interpolating from unaffected conditions upstream and downstream.

3.1.2 Channel Width

Accurate data regarding channel width is very important for fish passage road crossing design, because channel width values are used directly in the culvert sizing process for the stream simulation and active channel design methods. The active channel width is the width of the channel at the ordinary high water level (OHW), and it delineates the highest water level regularly experienced for a given water body. The active channel width is typically determined in the field and is commonly associated with any of the following:

- the bank elevation at which cleanly scoured substrate of the stream ends and terrestrial vegetation begins (Figure 3-2)
- natural line impressed on the bank
- presence of wood debris.

The bankfull width is defined as the point on a streambank at which overflow into the floodplain begins. The floodplain is a relatively flat area adjacent to the channel constructed by the stream. If the floodplain is absent or poorly defined, other indicators may identify bankfull, such as:

- a change in vegetation, slope or topographic breaks along the bank (Figure 3-2)
- a change in the particle size of bank material
- undercuts in the bank.

The recurrence interval of the active channel flow is slightly more frequent than the 2-year flood and varies from stream to stream. If possible, field determination of bankfull event should be calibrated to known stream flows or to regional relationships between bankfull flow and watershed drainage area.

The values of active channel width and bankfull channel width used for design are determined by averaging the widths of several measurements. It is common to take five or more measurements to determine the average active channel and bankfull widths. The measurements should be taken on straight reaches outside the influence of any existing culvert or structures.

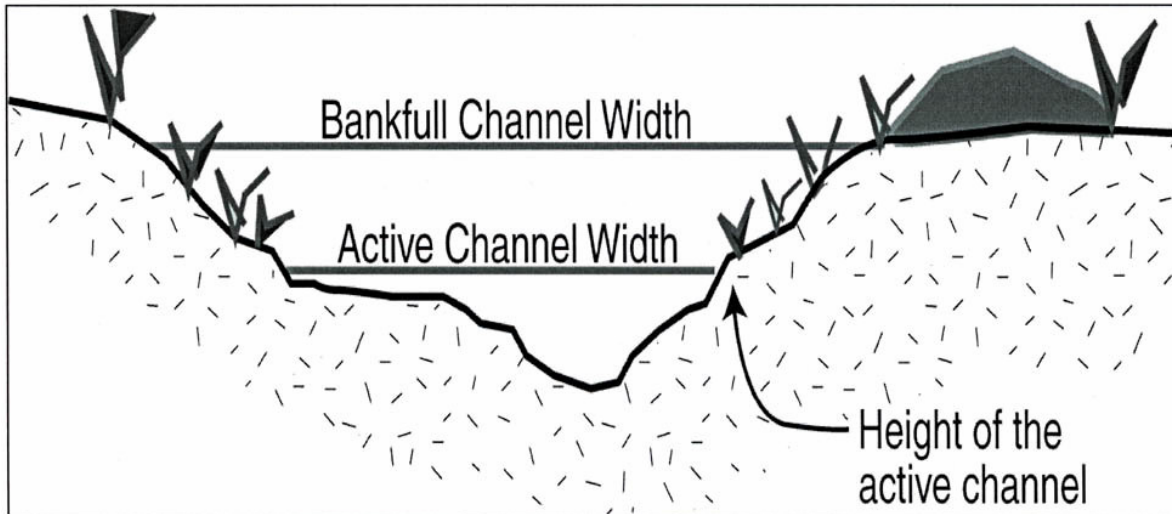


Figure 3-2. Field characteristics for identifying active channel and bankfull channel margins. (CDFG 2003)

Documentation of this data may be facilitated (or may have been previously prepared for planning purposes) using the active channel portion (page 7) of the form for Caltrans Fish Passage Data Collection - Second Pass Survey Information. Additional information describing the identification and measurement of the active channel width can be found in Part IX of the California Salmonid Stream Habitat Restoration Manual (CDFG 2003) at <http://www.dfg.ca.gov/nafwb/index.html>.

3.1.3 Channel Cross-sections

In Section 3.1.1., it was noted that longitudinal profile data is essential to the accuracy of a fish passage road crossing design. Similarly, good cross-section data is important. Cross-sections should be completed for a minimum of five locations: one each at the tailwater control points upstream and downstream of the crossing location, one each at a distance about 5 feet above and 5 feet below the crossing structure, and one at the structure centerpoint. Cross sections should note the elevation of both the active channel margin and the bankfull channel margin.

Most flow analysis models used for road crossing design and analysis are one-dimensional models. These models are based on equations that assume the streamlines are all parallel to each other. Therefore, cross-sections to be used in channel analysis computations should represent geometry which is "normal" (perpendicular) to stream lines. In cases where the stream is undergoing a significant expansion or contraction in width, a 'dog-leg' in the cross-section will provide more accurate analytical results (Figure 3-3). The breakpoint in the section should occur at the deepest point in the channel.

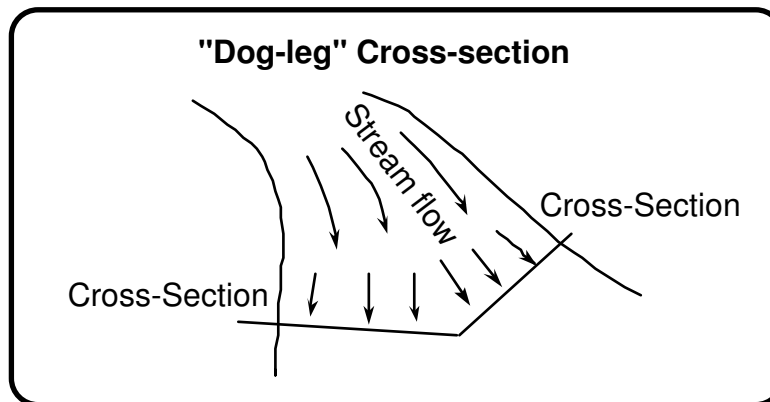


Figure 3-3. Illustration of a dog-leg cross-section.

3.1.4 Channel Roughness

Flow analysis models give considerable weight to the assigned roughness coefficient (Manning's n value) to estimate discharge, velocity and flow depth conditions. Guidelines for traditional culvert design frequently focus only on the roughness characteristics of the culvert material itself. In comparison, fish passage road crossing design will commonly assess flow conditions in the channel above and below the crossing. This will commonly require a more detailed assessment of roughness conditions than occurs with traditional culvert design.

The recommended method for assessing channel roughness subdivides a given cross-section into subsections for varying roughness elements and geometric characteristics. With the section subdivision, a roughness coefficient should be assigned for each change in vegetation and geometry. Figure 3-4 shows an example cross-section in which the channel has been subdivided to simulate flow characteristics reasonably.

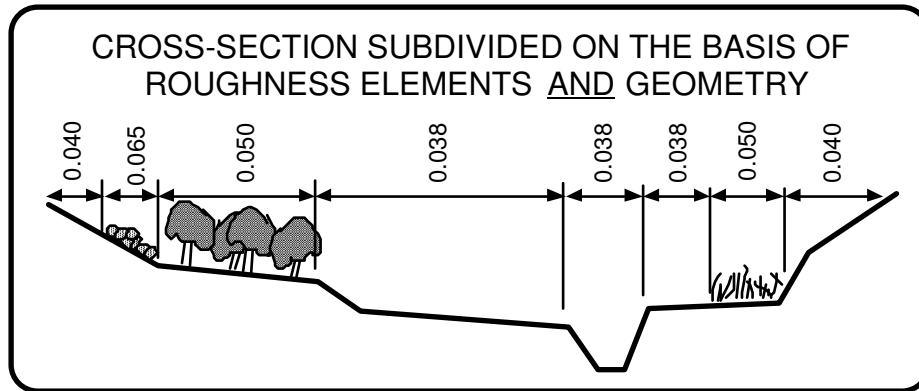


Figure 3-4. Example of subdividing a channel cross section to assign varying channel roughness values.

There are several guide tables that relate suggested roughness values to channel cover and configuration descriptions, including:

- FHWA publication HDS No. 3, "Design Charts for Open-Channel Flow"
- Chow's "Open Channel Hydraulics", which suggest 'normal' values along with a typical range of n-values
- FHWA Report No. FHWA-TS-84-204, "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains". This report presents a system in which several components contribute an additive amount to a total coefficient. The components considered include stability of channel, bank irregularities, vegetation, alignment of channel, and depth of flow.

Selection of roughness values for pipe or culvert structures may find the following guides especially helpful:

- FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts"
- HDM Table 851.2

3.1.5 Bed Load Composition

Bed load is characterized as sediment (silt, gravel, and rock debris) that moves by sliding or rolling very near the creek or river bed. Related to bed load, suspended load is the portion of the total sediment load that is created from upward momentum during turbulent flows. Because flow conditions can vary, particles comprising bed load may become suspended and later roll along the bed once again as part of the bed load. This means that the distinction and identification of bed load versus suspended load can be nebulous.

Given the variability in bed load composition, a project site must be investigated for potential types and sizes. A bed load consisting of just silt and gravel can cause capacity problems in any culvert. If baffles are introduced into an existing culvert, capacity can be further compromised due to increases accumulation of this bed load. Also, the silt and gravel can cause abrasion of concrete and steel baffles over time, leading them to failure. Of course, this problem of damage is exasperated with rock and large organic debris. Depending on flow conditions, larger rocks and woody debris can be carried as bed load, which could level a series of baffles. If this occurred, not only would fish passage be affected, but the rocks and debris can be lodged inside

a culvert and create objectionable backwaters.

Therefore, bed load is an important consideration in developing a sound fish passage design. For both rehabilitated and replacement culverts, excessive bed load can cause capacity problems and structural damage. With this possibility of high bed load volume and/or size, fish passage design strategy may change, for instance, a culvert may require being replaced instead of being rehabilitated with baffles solely because of bed load concerns. A new culvert could be sized properly to address fish movement, as well as convey the higher flood flows with potential bed load deposition.

3.1.6 Stream Stability

Fish passage road crossing project analysis should include a geomorphic processes assessment to ensure the long term viability of a structure. FHWA's "Stream Stability at Highway Structures (HEC 20)" outlines a detailed process for assessing stream stability. This is typically outside the requirements of a traditional culvert project. However, since strategies for fish passage road crossing design are tied closely to emulating stream conditions, the following qualitative assessments should be made prior to the design process.

- **Land Use Changes** – Large scale changes to a watershed can have dramatic impacts to the sediment yield of the watershed. Examples of large scale changes can be the result of logging activities, forest fires, dams, or development activities. Changes to the hydrology of a watershed will affect sediment yield and can impact both the vertical and lateral stability of a stream. Because of possible impacts to stream stability, it is important to develop a "long profile," as described in Chapter 5, in order to identify channel forming influences.
- **Lateral Stability** – Lateral stability of a stream can impact a crossing structure by increasing bank erosion. Bank failure can undermine structural components and road prisms and can block flow through the culvert, significantly reducing the conveyance capacity. Lateral instability can be the result of natural processes but can also be the result of a culvert that is not aligned properly. A qualitative assessment should be conducted early in the process to for past or anticipated changes in the stream alignment. Historical aerial photos and interviews with local residents are good tools for this qualitative assessment. In cases showing high risk of channel migration, selection of a bridge road crossing may provide the best means to accommodate the risk.
- **Vertical Stability** – Vertical instabilities refer to a stream going through and aggradation (sediment accumulation) or degradation (sediment scour). A channel experiencing aggradation is typically expected to widen ultimately resulting in shallower flow depths. Aggradation can also reduce the effective flow area beneath a crossing structure. A channel going through degradation may eventually threaten structural components of a crossing structure. A common example of a degradation process is the presence of a headcut. Headcuts can look like tiny water falls and progress upstream. If unaccounted for a headcut may progress upstream and create a perched outfall. Even worse, if a bottomless culvert is installed, a headcut can progress through the culvert, threatening both the bottomless culvert and upstream habitat.

If stream instabilities are identified during this qualitative assessment, a more thorough evaluation may be required. Ultimately, engineered stabilizing measures may be required to ensure long-term functionality of the fish passage structure.

The replacement of existing culverts may result in temporary channel instability:

- In cases involving replacement of an undersized culvert, an evaluation will be necessary regarding the potential for upstream bed instability following replacement. The upstream instability potential is created due to the fact that, at high flows, undersized culverts create a backwater, which increases the probability that bed material will be deposited in the channel upstream. With elimination of the backwater condition, the upstream channel profile will be expected to lower over time. Channel bed and bank protection measures may be required.
- Outlet velocities from traditional road crossings can be potentially erosive. Of particular concern to fish passage is the potential for creation of a scour hole immediately downstream from the road crossing. In severe cases, a road crossing may become perched above the stream bed due to these erosive forces. Design processes are presented in this manual which provide rehabilitation of these degraded conditions and protect against future degradation. Sites with banks or beds susceptible to erosion may require special consideration. Indications of poor channel stability may suggest the use of a bridge as the road crossing type.

3.2 Hydrology for Road Crossing Design

California exhibits a very diverse environment in terms of hydrologic characteristics, ranging from extremely arid to near rain-forest conditions. This diversity presents a challenge to the design of highway drainage systems. Traditional culvert design methods focus on peak flow conditions that occur on an infrequent basis, say, every 25 or 100 years. Fish passage road crossings, on the other hand, also emphasize evaluation of flow conditions that occur every year when the target populations are present.

3.2.1 Overview of Hydrologic Methods

This section presents three methods that can be used to calculate high and low design flows for fish passage road crossings. Design flows can be determined using 1) local stream gage data to estimate annual exceedance factors; 2) USGS regional regression equations; and 3) the Natural Resources Conservation Service (NRCS) Urban Hydrology for Small Watersheds Technical Release 55 (TR-55). A detailed description and example calculation for a fourth method, the Rational Method, can be found in the Caltrans Highway Drainage Design course materials.

A general discussion on the hydrologic process is not presented in this section because numerous textbooks discuss the hydrologic process in detail. As a refresher, a good discussion on hydrology is presented in Chapter 810 of the Caltrans Highway Design Manual (HDM). Additional references that may be useful include:

- The Natural Resources Conservation Service, Urban Hydrology for Small Watersheds Technical Release 55 (TR-55). This document can be found at the U.S. Department of Agriculture's website <http://www.info.usda.gov/CED/>
- Rantz, S.E., 1969, Mean annual precipitation in the California region: U.S. Geological Survey Open-File Map (Reprinted 1972, 1975).
- Miller, J.F., Frederick, R.H., and Tracey, R.J., 1973, Precipitation – Frequency Atlas of the Western United States, Volume XI – California, NOAA.

3.2.2 Selecting the Appropriate Method

In most instances, watershed characteristics control which hydrologic method is used for analysis. Contributing to the method selection is the available information for the watershed.

For instance, it is unlikely that a stream gage would be located at or even near the stream crossing under consideration. Gage data is typically recorded on large streams where stream crossings have already been designed and constructed.

Table 3-1 below provides guidance on which method is appropriate to use based on the watershed characteristics and available information.

Table 3-1. Guidance on selection of hydrologic methods.

METHOD	ASSUMPTIONS	DATA NEEDED
Exceedance**	<ul style="list-style-type: none"> At least five years of recorded daily average flows, and preferably more than ten-years (do not need to be consecutive years) Drainage area less than 129.5 km² (50 mi²) (preferably less than 25.9 km² (10 mi²)) Unregulated flows (no upstream impoundment or water diversions) 	<ul style="list-style-type: none"> Gage Data from nearby stream Drainage area of both watersheds
Regional Regression*	<ul style="list-style-type: none"> Catchment area limit varies by region Ungaged channel Basin not located on floor of Sacramento and San Joaquin Valleys Peak discharge value for flow under natural conditions unaffected by urban development and little or no regulation by lakes or reservoirs 	<ul style="list-style-type: none"> Drainage area Mean annual precipitation Altitude Index
TR-55*	<ul style="list-style-type: none"> Small or midsize catchment (< 8 km² (< 3.1 mi²)) Concentration time range from 0.1 to 10-hour (tabular hydrograph method limit < 2 hour) Runoff is overland and channel flow Simplified channel routing Negligible channel storage 	<ul style="list-style-type: none"> 24-hour rainfall Rainfall distribution Runoff curve number Concentration time Drainage area

*Refer to the Caltrans Highway Design Manual for further information

**Refer to the California Salmonid Stream Habitat Restoration Manual for further information

CDFG and NOAA Fisheries recommend that if stream gage data is available, the exceedance flow method is the preferred option to calculate the fish passage flows (CDFG 2002, NOAA-SWR 2001). Table 3-2 shows the percentages of the annual exceedance flow recommended to be used for each of five classifications of fish species listed in the criteria. If stream gage data is not available, then the discharge for the 2-year flow should be calculated using either the regional regression or TR-55 methods, and a percentage of the 2-year is used for high fish passage design flow, as shown in Table 3-2.

Table 3-2. Factors for determining high fish passage flow rate. (CDFG 2002)

Species/Life Stage	Percentage of Annual Exceedance Flow	Percentage of 2-year Recurrence Interval
Adult Anadromous Salmonids	1%	50%
Adult Non-Anadromous Salmonids	5%	30%
Juvenile Salmonids	10%	10%
Native Non-Salmonids	5%	30%
Non-Native Species	10%	10%

In determining lower fish passage flow, again, if stream gage data is available, the exceedance

flow method should be used to calculate the flow, applying a percentage factor to the average annual exceedance flow as shown in Table 3-3. If gage data is not available, then the alternate minimum flow should be used.

Table 3-3. Factors for determining low fish passage flow rate. (CDFG 2002)

Species/Life Stage	Exceedance Flow	Default Minimum Flow
		(ft ³ /s)
Adult Anadromous Salmonids	50%	3
Adult Non-Anadromous Salmonids	90%	2
Juvenile Salmonids	95%	1
Native Non-Salmonids	90%	1
Non-Native Species	90%	1

The following sections provide more information for determining the high and low fish passage flow rates using each of the exceedance flow, regional regression, and TR-55 methods.

3.2.3 Exceedance Flow Rates using Gage Data

Figure 3-5 shows a typical distribution of flow data representing an average annual flow. The figure points out three exceedance flow rates and notes how they frequently relate to fish passage concerns. The 95% exceedance flow, for example, is the flow rate which is exceeded 95% of the time on an annual basis; the lowest 5% of flow rates below this threshold may be so low as to result in flow depths that are too shallow for fish to swim in. It is important to note that these **average annual exceedance flows rates are not to be confused with exceedance flow probabilities**, which involve statistical analysis using **annual peak flows**.

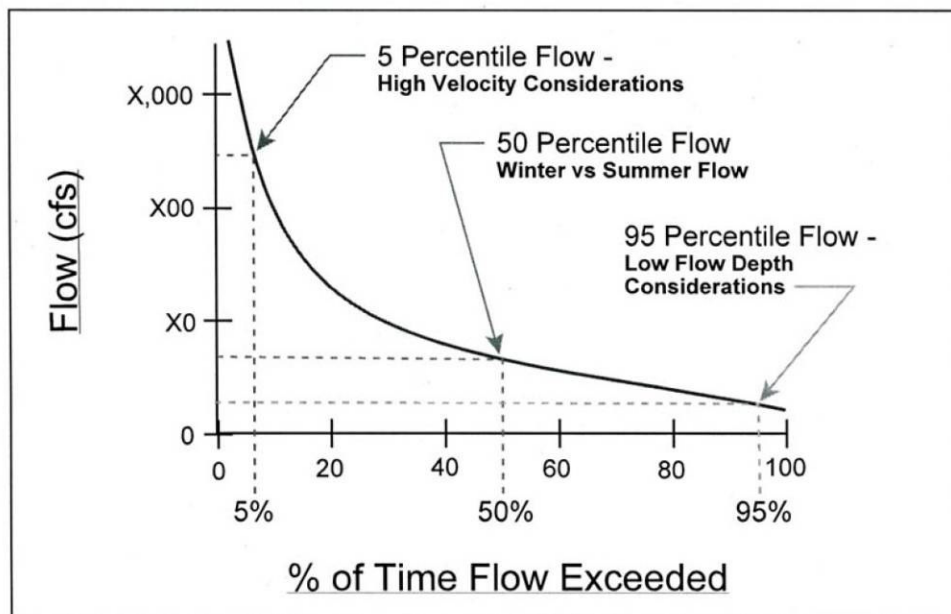


Figure 3-6. A typical flow duration curve for average annual flow data. (CDFG 2003)

Identifying exceedance flows requires obtaining average daily stream flow data. If the stream flow rate is known based on gage data collected for that stream, then the crossing should be sized based on that data. Often times, a crossing is to be designed on a stream where gage data is not available. However, if a nearby stream has gage data and the stream where the crossing is to be designed has similar watershed characteristics, then the available gage data can be adjusted and used for design. District Hydraulics should be contacted to assess comparable watershed characteristics. The method presented below describes how to adjust nearby stream gage data to estimate the peak stream flow rate. The following method was abstracted from Part IX of the California Salmonid Stream Habitat Restoration Manual (CDFG 2003). For more information please reference that Manual.

1. Flow records for nearby streams should be acquired from the USGS and/or the California Department of Water Resources. The information must meet the following requirements:
 - At least 5-years of recorded daily average flows, and preferably more than 10-years (do not need to be consecutive years)
 - A drainage area less than 50 square miles (130 km²), and preferably less than 10 square miles (26 km²)
 - Unregulated flows (no upstream impoundment or water diversions). If feasible, use several gaged streams to determine which ones have flow characteristics that best resemble stream flows observed throughout the project area.
2. Rank the flows from highest to lowest (a rank of $i=1$ given to the highest flow). The lowest flow will have a rank of n , which equals the total number of flows considered. To identify rank associated with a particular exceedance flow, such as the 50 percent and 1 percent exceedance flows ($i_{50\%}$ and $i_{1\%}$) respectively, use the following equations:

$$i_{50\%} = 0.50(n+1) \quad i_{1\%} = 0.01(n+1)$$

3. Round values to the nearest whole number. The flows corresponding to those ranks are the 50 percent and 1 percent exceedance flows for the gaged stream.
4. To apply these flows to the ungaged stream, multiply the flows obtained in the above step, $Q_{50\%}$ and $Q_{1\%}$, by the ratio of the gaged stream's drainage area (DA) to the drainage area of the ungaged stream at the stream crossing. Multiplying by this ratio adjusts for the differences in drainage area between watersheds.

Other methods for determining exceedance flows for ungaged streams can also be used. These methods typically take into account differences in precipitation between watersheds.

When flows from several different gaging stations are available, use knowledge of the local hydrology and rainfall patterns to decide which one offers the best estimate. For inventory and assessment purposes, the method described above is often sufficient. More detailed or accurate flow measurement techniques may be necessary in the design of new or replacement stream crossings.

Other things to consider when using gage data includes:

- This method is limited in a number of ways, one of which is the fact that it only considers a narrow time frame in the life time of the stream crossing. For example, stream flow data may have only been collected during a drought. This would result in sizing a fish passage that is too small. Inversely, the fish passage could be sized too large if the gage data was taken during years of high rainfall.

- A second limitation of this method is the transfer of stream flow data from one watershed to another. Although the watersheds may be near each other, there will still be differences between the two. Cover, detention, soil type, slope, and even rainfall could vary between the two watersheds. Careful inspection of the two watersheds should be conducted to determine if it is reasonable to transfer the data.

An example calculation of fish passage flow rates using the exceedance flow rate method is provided in Appendix E.

3.2.4 Regional Regression Equations

Regional Regression equations have been developed for the state of California to estimate the peak discharge for a watershed for recurrence intervals of 2, 5, 10, 25, 50, and 100 years. The state is divided into six hydrologic regions and each region has specifically derived equations unique to that region. A map showing the different regions is shown in Figure 3-7. The parameters for the equations include drainage area (A), in square miles; mean annual precipitation (P), in inches; and an altitude index (H), which is the average altitudes in thousands of feet at the points along the main channel at 10 percent, and 85 percent of the distances from the site to the divide (USGS 1993).

Area and altitude index are determined from a topographic map, and mean annual precipitation is determined from a map in Rantz (1969). The USGS provides non-proprietary software that may be used to calculate the flows using the regression equations. The software is available at their website, www.usgs.gov, and is called the National Flood Frequency Program (NFF). Table 3-x shows the equations used to calculate the design flow rates for the six hydrologic regions in California.



Figure 3-7. Flood-frequency region map for California. (USGS 2004)

Table 3-4. Regional regression equations for the six regions of California. (USGS 1993)

North Coast Region

$$Q_2 = 3.52A^{0.90}P^{0.89}H^{-0.47}$$

$$Q_5 = 5.04A^{0.89}P^{0.91}H^{-0.35}$$

$$Q_{10} = 6.21A^{0.88}P^{0.93}H^{-0.27}$$

$$Q_{25} = 7.64A^{0.87}P^{0.94}H^{-0.17}$$

$$Q_{50} = 8.57A^{0.87}P^{0.96}H^{-0.08}$$

$$Q_{100} = 9.23A^{0.87}P^{0.97}$$

In the North Coast region, use a minimum value of 1.0 for the altitude index (H).

Northeast Region

$$Q_2 = 22A^{0.40}$$

$$Q_5 = 46A^{0.45}$$

$$Q_{10} = 61A^{0.49}$$

$$Q_{25} = 84A^{0.54}$$

$$Q_{50} = 103A^{0.57}$$

$$Q_{100} = 125A^{0.59}$$

Maximum drainage basin is 40 km² for the Northeast region.

Sierra Region

$$Q_2 = 0.24A^{0.88}P^{1.58}H^{-0.80}$$

$$Q_5 = 1.20A^{0.82}P^{1.37}H^{-0.64}$$

$$Q_{10} = 2.63A^{0.80}P^{1.25}H^{-0.58}$$

$$Q_{25} = 6.55A^{0.79}P^{1.12}H^{-0.52}$$

$$Q_{50} = 10.4A^{0.89}P^{1.03}H^{-0.41}$$

$$Q_{100} = 15.7A^{0.77}P^{1.02}H^{-0.43}$$

Central Coast Region

$$Q_2 = 0.0061A^{0.92}P^{2.54}H^{-1.10}$$

$$Q_5 = 0.118A^{0.91}P^{1.95}H^{-0.79}$$

$$Q_{10} = 0.583A^{0.90}P^{1.61}H^{-0.64}$$

$$Q_{25} = 2.91A^{0.89}P^{1.26}H^{-0.50}$$

$$Q_{50} = 8.20A^{0.89}P^{1.03}H^{-0.41}$$

$$Q_{100} = 19.7A^{0.88}P^{0.84}H^{-0.33}$$

South Coast Region

$$Q_2 = 0.14A^{0.72}P^{1.62}$$

$$Q_5 = 0.40A^{0.77}P^{1.69}$$

$$Q_{10} = 0.63A^{0.79}P^{1.75}$$

$$Q_{25} = 1.10A^{0.81}P^{1.81}$$

$$Q_{50} = 1.50A^{0.82}P^{1.85}$$

$$Q_{100} = 1.95A^{0.83}P^{1.87}$$

South Lahontan-Colorado Desert Region

$$Q_2 = 7.3A^{0.30}$$

$$Q_5 = 53A^{0.44}$$

$$Q_{10} = 150A^{0.53}$$

$$Q_{25} = 410A^{0.63}$$

$$Q_{50} = 700A^{0.68}$$

$$Q_{100} = 1080A^{0.71}$$

Maximum drainage basin is 40 km² for the South Lahontan-Colorado Desert regions.

Where:

A = Drainage area, mi²

P = Precipitation, inches

H = altitude index

Other things to consider when using the Regional Regression equations include:

- Ground conditions play a significant role in the peak flow rate of a stream. Bare ground with little infiltration and a steep slope will result in a higher peak flow rate because water will reach the point of interest faster than the same area that has lush ground cover, absorbent soils, and a flat slope.
- Drainage area and altitude index are easily calculated from a topographic map. Mean annual precipitation, on the other hand, is a general estimate for an area and not specific to a particular watershed. Rainfall amounts collected at various gages throughout a region are extrapolated over that region to get isohyets, or lines of equal rainfall. Mean annual precipitation for a region is based on these isohyets that are drawn from information collected over a number of years. A number of publications can be consulted for further discussion on the derivation and applicability of mean annual precipitation.
- Inherent in the regression equations are errors of estimate. According to the USGS, the standard error of estimate for the California regression equations ranges from 60 to 100 percent.
- Regression equations should be used when little is known about the watershed. If sufficient information about the watershed is available, use of the other methods described in this section is recommended for analysis.
- For more information of the development and use of regression equations refer to the U.S. Geological Survey Water-Resources Investigations Report 94-4002.

An example calculation of fish passage flow rates using the regional regression equation method is provided in Appendix E.

3.2.5 TR-55 Method

The TR-55 method presents simplified procedures for estimating runoff and peak discharges in small watersheds. The method is geared towards estimating runoff in urban and urbanizing watersheds; however, the procedures apply to any small watershed in which certain limitations are met.

The method begins with the assumption that rainfall is uniformly imposed on the watershed over a specified time distribution. TR-55 includes four regional rainfall time distributions for a 24-hour period. The rainfall distributions were designed to contain the intensity of any duration of rainfall for the frequency of the event chosen.

Mass rainfall is converted to mass runoff by using a runoff curve number (CN). CN is based on soils, interception, and surface storage. Runoff is then transformed into a hydrograph by using unit hydrograph theory and routing procedures that depend on runoff travel time through segments of the watershed (TR-55 1986).

Three steps are performed to calculate the peak discharge of a drainage area. The three steps are to calculate the Q in inches, calculate the time of concentration in hours, and then calculate the peak discharge. The three steps are described in the following sub-sections.

The TR-55 method is used for a single hydrologically homogenous watershed. If the watershed is heterogeneous, made up of several homogenous subareas, then the TR-55 publication should be consulted. TR-55 also addresses how to use detention basins to reduce the peak flow rate of an urbanizing watershed.

An example calculation of fish passage flow rates using the TR-55 method is provided in Appendix E.

3.3 Basic Hydraulics for Fish Passage Road Crossing Design

Basic hydraulic principles of open channel flow must be understood for application in fish passage road crossing design, as well as for other aspects of highway drainage. Flows in the natural stream channels above and below a road crossing are governed by the principles of open channel flow. Flows through culverts and under bridges also are governed by these basic principles, as long as there is a free surface.

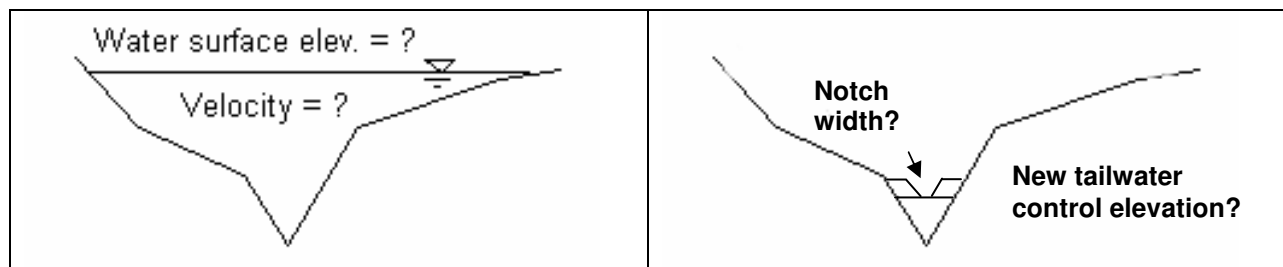
The following materials are adapted largely from a Caltrans Highway Drainage Design Course developed as an aid for the design of traditional road crossings where fish passage is not an issue. Materials have been added or deleted as appropriate to make them relevant to fish passage road crossings. There are numerous useful published references that can provide additional background on the principles of open channel flow. Some of these references are:

- Caltrans Highway Design Manual, Chapter 800,
- "Open Channel Hydraulics" by Ven Te Chow,
- "Handbook of Hydraulics" by Brater and King,
- FHWA Hydraulic Design Series No. 4, "Design of Roadside Channels",
- FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts",

Principles of open channel flow are applied to fish passage road crossings from the perspective of both analysis and design. Channels usually are analyzed for the purpose of determining the characteristics of the stream flow. In the analysis of existing channels, the designer is working with geometric parameters that are fixed. Analysis is performed using the fixed stream channel parameters to determine the relationship between stream discharge, depth or water surface elevation, and flow velocities.

When designing an open channel feature, on the other hand, the designer controls many of the geometric parameters of the feature. Design of the feature typically involves a trial and error process, where a trial configuration is assumed and an analysis is made to affirm or negate the assumed configuration. This process is repeated until the feature design is shown to satisfy the specified design criteria.

The design of fish passage road crossing facilities often involves analysis of the existing stream channel. Depending on the conditions of the project site, it may also be necessary to design open channel features to enhance fish passage conditions. Schematically, the contrasts between analysis and design of channels are illustrated by Figure 3-8.



Analysis	Design
Given: Discharge (usually), channel geometry, slope, roughness characteristics.	Given: Discharge, certain design constraints (min. depth, max. velocity, range of slopes).
Find: Velocity and discharge vs. water surface elevation relationship.	Find: Optimum geometry for fish passage enhancements

Figure 3-8. Contrasting elements of open channel analysis versus open channel design.

3.3.1 Types of Flow

Open channel flow frequently classifies flow according to changes in flow depth with respect to time and space.

Steady flow is defined as flow in which there is no variation of depth of flow with respect to time. This definition is extended to mean that there is no variation in the discharge with respect to time. The opposite of steady flow is *unsteady flow*, which often is referred to as dynamic flow. Most of the common channel analysis procedures assume steady flow.

Uniform flow is used to indicate the depth of flow is the same at every section of the channel. The opposite of uniform flow is *varied flow*, in which the depth of flow is not the same at every section of the channel. *Rapidly varied flow* is classified by a sudden change in water depth over a comparatively short distance. This type of flow frequently occurs as the result of significant changes in channel configuration, such as occurs with flow through orifice, flow over weirs, or sudden changes in channel slope. Varied flow that changes depth over longer distances is classified as *gradually varied flow*.

Steady, uniform flow is the assumed condition for many open channel analyses. The simplest procedures for determining flow characteristics use one-dimensional uniform flow models. These models assume the velocity vectors within a channel are all more or less parallel with one another; with no horizontal or vertical transfer of flow taking place. For standard, non-fish passage culvert design concerned primarily with headwater depth and outlet velocity, these one-dimensional flow models usually provide the level of detail necessary for analysis.

Culvert and road crossing designs frequently add features such as tailwater control weirs and baffles that are likely to result in rapidly varied flow. In cases where the hydraulic conditions of water depth and velocity must be assessed in the vicinity of such structures, it may be advisable to refine channel and backwater analyses through the use of two-dimensional models.

3.3.2 Flow Regimes

The specific energy (E) of a channel section may be represented by the sum of the flow depth and the velocity head associated with that flow depth. In equation form,

$$E = y + \frac{v^2}{2g} \quad \text{Equation 3-1}$$

where: E = specific energy (m),
y = depth of flow (m),
V = velocity of flow (m/s),
g = acceleration due to gravity; 9.81 (m/s²)

For a given specific energy, there are two possible depths at which the open channel flow may occur. One depth occurs at a low stage in the supercritical flow regime, and the other occurs at a high stage in the subcritical flow regime. At the point where specific energy is a minimum, the flow is at a critical state of flow, and hence the associated depth is termed the critical depth (d_c). The specific energy diagram for open channel flow shown as Figure 3-9 illustrates the relations between supercritical, critical, and subcritical flow.

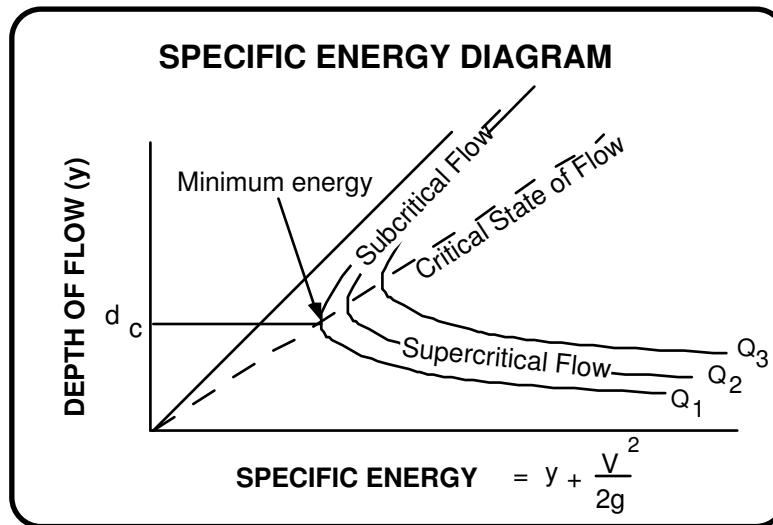


Figure 3-9. The specific energy diagram for open channel flow.

The significance of the specific energy diagram to fish passage is to note that the conditions of subcritical flow are generally significantly more favorable to fish passage than supercritical flow. For a given energy level, the subcritical flow condition has a greater flow depth and a lower velocity than its supercritical counterpart. Measures which promote the flow of a stream or road crossing structure to operate under subcritical flow will promote fish passage.

The flow regime occurring in natural stream channels is largely dependent on the slope of the channel. On a much more localized level, the slope of a road crossing structure will also factor into the flow regime.

- **Supercritical flow** - In mountainous or hilly terrain, the stream bed is often very steep and will result in supercritical flow. Similarly, in the localized realm of a road crossing structure, a culvert installed with a steep slope will be prone to supercritical flow. Supercritical flow is also called steep slope regime.
- **Subcritical flow** - Subcritical flow commonly occurs when the stream bed slope is relatively flat; therefore it appears in foothills, alluvial areas, and most areas outside of mountains. Culverts placed at a mild slope will commonly exhibit subcritical flow as well. Subcritical flow is also called mild slope regime.
- **Critical flow** - For a given discharge and channel geometry, there is one depth of flow that will result in minimum energy required to maintain the given discharge. The depth usually is referred to as critical depth (d_c). Critical flow is the threshold between supercritical and subcritical flow, and it commonly occurs where there are sudden changes in channel configuration, such as at weirs or perched culvert outfalls.

Determination of whether a stream is flowing in the supercritical, subcritical, or critical flow regime can be determined by calculation of the Froude number:

$$Fr = \frac{Q^2 T}{g A^3} \quad \text{Equation 3-2}$$

where: Q = discharge (m^3/s),
T = top width of free surface (m),
g = gravitational acceleration, and
A = cross-sectional area of flow (m^2).

and: Fr = 1 is critical flow,
Fr > 1 is supercritical flow, and
Fr < 1 = subcritical flow,

3.3.3 Average Velocity

In any channel section, the discharge Q is related to the cross-sectional area through the equation,

$$Q = VA \quad \text{Equation 3-3}$$

where: Q = rate of discharge in cubic meters per second (m^3/s),
V = average velocity of the water in meters per second (m/s), and
A = cross-sectional area normal to the direction of flow in the channel or conduit, in square meters (m^2).

The average velocity V is a defined entity; that is, V is simply defined as the actual discharge Q divided by the actual cross-sectional area A. Because open channel flow involves both a free surface and friction along the channel perimeter, the actual velocities in a channel are not uniformly distributed in the channel section. The maximum velocity usually occurs slightly below the free surface, while the minimum velocities are typically at the boundary layer of channel flow. Figure 3-10 illustrates the typical velocity distribution in channels of various cross-sectional shape. Velocity distribution will also be affected by factors such as the presence of bends and the degree of roughness at the channel perimeter.

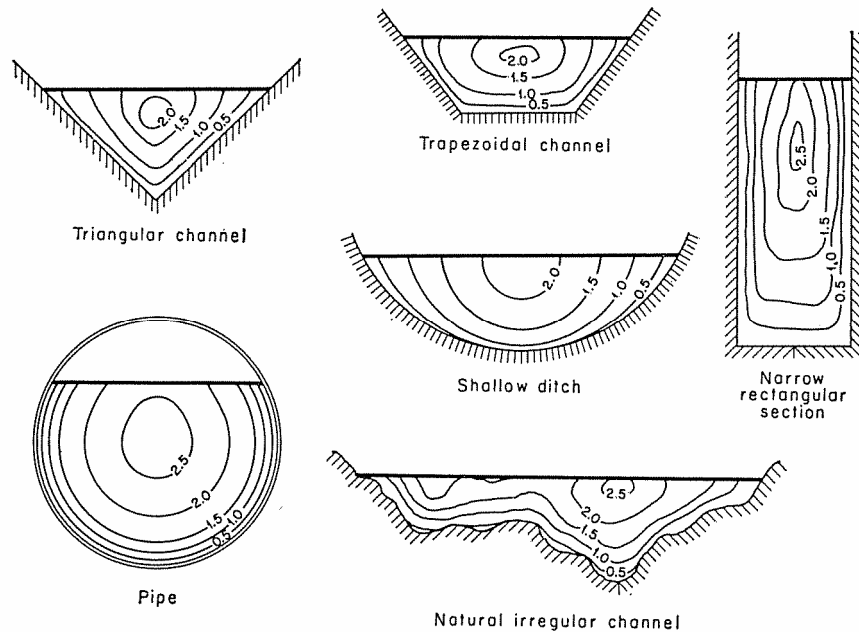


Figure 3-10. Typical velocity distributions in various channel sections, noting lines of equal velocity. (Chow 1958)

Fish passage criteria for road crossings often establish a maximum velocity that should not be exceeded during fish passage flows. These criteria are presented in terms of the average velocity within the channel, as calculated by Manning's equation or other flow analysis models. It should be noted there is an inherent safety factor in designing to the average velocity, since actual flow velocities at the boundary layer of the channel are likely to be lower than the average velocity.

3.3.4 Manning's Equation

Manning's equation is a widely-used uniform flow formula for determining flow characteristics on open channels. The basic form of Manning's equation is:

$$V = \left[\frac{R^{2/3}}{n} \right] S^{1/2} \quad \text{Equation 3-4}$$

where: V = velocity of flow (m/s),
 R = hydraulic radius (m) = cross-sectional area of flow A (m^2) divided by the wetted perimeter P (m),
 S = longitudinal slope of the water surface (m/m),
 n = the roughness coefficient (dimensionless); (Manning's n).

Merging the original Manning's equation with the continuity equation relationship of $Q = VA$ yields another popular form of the Manning's equation:

$$Q = \left[\frac{A R^{2/3}}{n} \right] S^{1/2} \quad \text{Equation 3-5}$$

In either form, it is worth noting that the velocity of discharge is inversely related to the roughness coefficient, commonly referred to as the Manning's n value. An example of the significance of this relationship is seen in the fact that, for a channel of given slope and geometry, the velocity will be cut in half if the roughness coefficient is doubled. Very generally, a channel lined with grass and weeds, with little or no brush (n = 0.30 to 0.35) would have a velocity about twice as fast the same discharge flowing through the same channel lined entirely with dense willows (n = 0.060 to 0.080). Measures that increase roughness within a channel used for fish passage can have a tremendous benefit in reducing velocities to levels the fish can negotiate.

3.3.5 Normal Depth Calculations

Manning's equation can be used to evaluate normal flow conditions in an open channel. Normal flow assumes that the water is neither accelerating or decelerating, and that it has constant velocity and depth. While this is a gross simplification of natural stream conditions, the calculation of normal depth provides a starting point for determining the actual depth for a given set of stream conditions. It can be used to estimate the depth of water at the tailwater control point under various flows, such as the low and high fish passage flows.

The difficulty of calculating the normal depth for a given discharge is that both the area A and the hydraulic radius R are determined using the unknown depth. One common solution is to assume several values of depth and plot the resulting discharge using Manning's equation. This type of graphical output is commonly called a stage discharge curve. The graph can subsequently be used to find the expected normal water depth for any selected discharge. An example is described below.

1. A cross-section of the tailwater control point is developed, noting the roughness coefficients for the various geographic sections. (Figure 3-11.)
2. A water elevation and associated water depth is assumed, and the discharge is calculated using Manning's equation. As intermediate steps, values for area A, wetted perimeter P, and hydraulic radius R are approximated using properties of triangles applied to each geometric section. Where the flow area crosses more than one roughness element, a composite roughness value is developed based on the weighted value of roughness in relation to the wetted perimeter (see section 3.4.x). The slope assumed for the calculation is the slope of the channel. [It may be helpful to provide a table here that shows the calculations and results for A, P, R and n.]
3. A tabulation and/or plot is made showing the computed discharges versus corresponding assumed water surface elevations. (Figure 3-12).
4. From the tabulation or plot, an approximate water surface elevation can be derived for any discharge desired. Since there is an assumption of uniform flow for this method, the associated water depth can be transferred up or down the stream to other locations not influenced by other hydraulic control points.

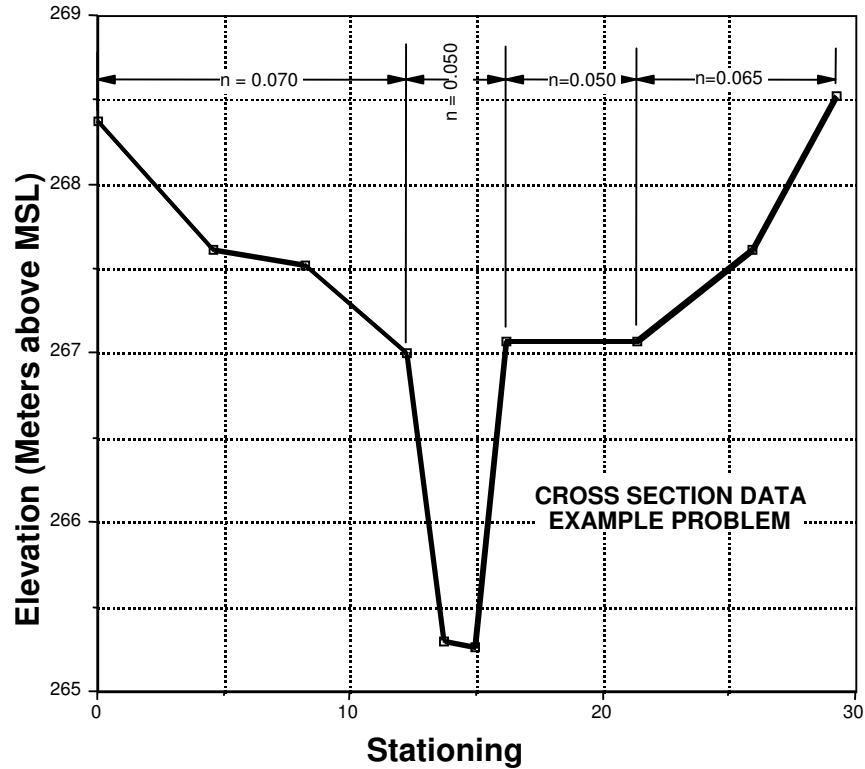


Figure 3-11. Example cross section data showing assigned roughness elements.

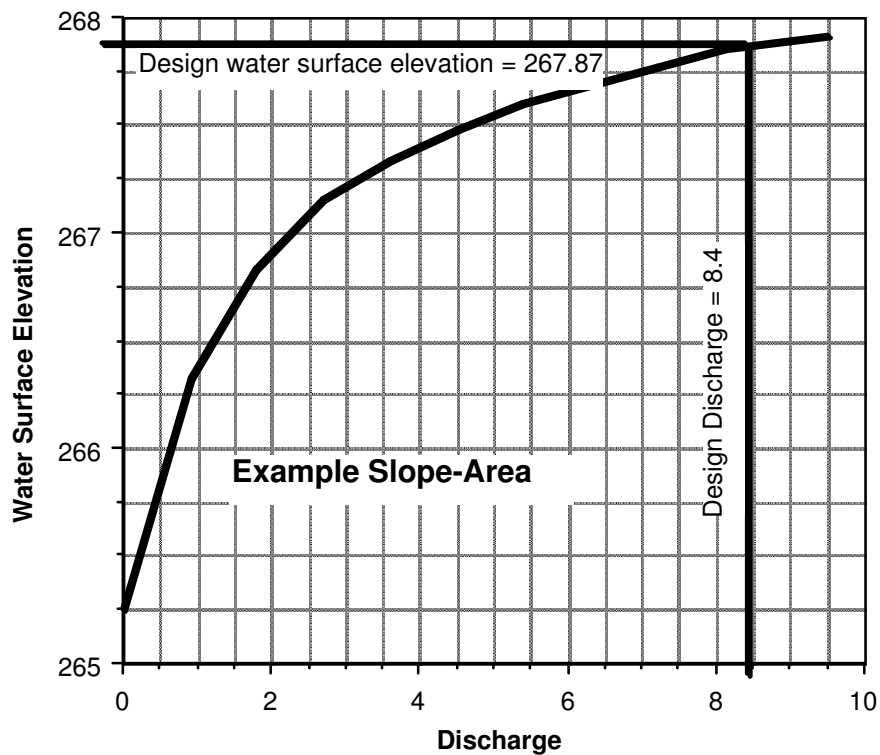


Figure 3-12. An example stage-discharge curve. The vertical and horizontal lines illustrate its use to determine flow depth at the selected design discharge.

This approach is simple and convenient and can be used effectively in most cases where uniform flow can be assumed. Examples of its application might include crossings where the channel slope is very consistent and there is no noticeable tailwater control point below the culvert.

3.3.6 Backwater Concepts

Flow in an open channel is rarely uniform. Even when discharge conditions are steady, the changes in channel slope and presence of hydraulic control points usually causes the flows to accelerate or decelerate along the channel. A common condition with fish passage road crossings is the deceleration that occurs as water approaches a tailwater control point, such as a rock weir constructed below a culvert outlet to increase water depths. Without the weir, the water might flow at the normal flow depth. With the weir in place, the water will slow down and deepen until it reaches the height which allows flow to pass over the weir crest at its critical depth. These control points are convenient starting points of known water depth where it becomes possible to determine upstream water depths using step-backwater methods.

Standard step-backwater methods involve the principle of conservation of energy as depicted in the Energy Equation, shown below for a channel of "small slope". Small slope usually is described as a slope less than 10%.

$$z_1 + y_1 + \alpha_1 \frac{V_1^2}{2g} = z_2 + y_2 + \alpha_2 \frac{V_2^2}{2g} + h_f \quad \text{Equation 3-6}$$

where:

- z_1 and z_2 = elevations (either arbitrary or above mean sea level) of the streambed at the upstream and downstream sections respectively (m);
- y_1 and y_2 = depths of flow at the upstream and downstream sections respectively (m);
- α_1 and α_2 = velocity distribution coefficients at the upstream and downstream sections respectively (dimensionless). The value of the velocity distribution coefficient depends, in large part, upon the subdivisions available in the cross-section for conveyance computations. It is derived exactly in most computer programs but is commonly assigned a value of 1.0 for hand calculations where the cross-section is divided into several subsections. The velocity distribution factor serves to accommodate the varying velocity across the cross-section.
- V_1 and V_2 = average velocity of flow at the upstream and downstream sections respectively (m/s);
- h_f = friction head loss from upstream to downstream (m). The friction head loss equals the distance between cross-sections multiplied by the slope of the energy line. Reference is made to texts on open channel flow for further discussions.

Other losses, such as eddy losses, expansion losses, and contraction losses may be considered in some cases.

g = acceleration due to gravity - 9.81 m/s².

Figure 3-10 is a graphical representation of the energy equation. By stepping from one cross-

section to the next in succession (in an upstream direction for sub-critical flow regimes and a downstream direction for super-critical flow regimes), one can define the profile of the water surface of an irregular channel for gradually varied flow. Thus the term step backwater computations.

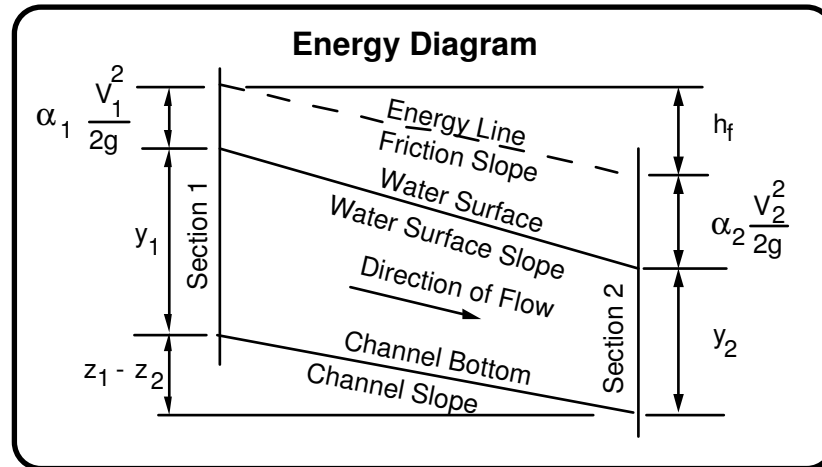


Figure 3-13. The energy diagram illustrating the principles of step-backwater analysis.

Even though they are direct calculations, manual computations of a standard step backwater procedure can be very tedious, especially for irregular cross-sections in natural streams. For this reason, several computer models have been developed to facilitate this analytical process.

3.3.7 Software for Fish Passage Road Crossing Hydraulics

There are several situations within this manual for which a backwater hydraulic calculation is recommended. The following paragraphs provide summaries of backwater model software commonly used for road crossing design.

A backwater model calculates subcritical hydraulic characteristics at a point in a channel based on the water surface just downstream plus the energy loss due to friction and change of channel shape between the two points. It may also calculate supercritical hydraulic characteristics and indicate which hydraulic flow regime exists at the point for the flow being analyzed.

The reasons for these analyses in road crossing design might be to do an analytical channel design, to compare an analytical design to a reference reach design, to evaluate whether the culvert inlet becomes submerged at a high flow, or to calculate the maximum capacity of a culvert. Designs that include long culverts and/or significant floodplain contractions should include backwater analyses to derive hydraulic slopes and shear stresses for comparison to the reference channel and to verify flow is subcritical throughout the project.

If a backwater analysis is conducted for a stream simulation design, verify that the Froude number in the stream simulation channel is similar to that in the reference channel. It is generally desirable that flows be subcritical at all flows up to at least the stable bed design flow.

Culvert hydraulic nomographs can be used for some of these purposes, but they are not available for embedded culverts with various bed materials, roughness, and depths, and therefore they generally are not suited for analysis of many fish passage culverts. Backwater calculations are

generally too intense for hand calculation. There are several computer backwater models available for the evaluation and design of culverts. They typically calculate headwater depth upstream of a culvert and/or average cross-section velocities, but not the velocity or turbulence in the pathways used by fish.

River Analysis Software (HEC-RAS) is a backwater model developed by US Army Corps of Engineers Hydrologic Engineering Center for any open channel flow calculations including bridges, culverts, divided flow, multiple culverts, and unsteady flow. It is available from Hydrologic Engineering Center and it is packaged into several commercial products as well. It can model complex channel hydraulics and calculate instream and floodplain velocities, shear stresses and more. Roughness can vary across the cross-section and with changing flow. It contains a module for culverts that includes beds inside of culverts but it does not specifically report the hydraulics inside the culvert. To work around that limitation, an open channel can be modeled with a lid over it, which acts as a culvert. Being a universal backwater model, it can directly compare the hydraulics of a natural channel to various culvert and bridge options. The software is well documented. It requires substantial data and knowledge of modeling.

FishXing (version 2.2) was developed by the US Forest Service. It is a backwater model with the capability of a variety of culvert shapes. It is the only model that directly examines hydraulics inside a culvert with a bed. The bed material must be flat in cross-section but can have a specified roughness other than the culvert walls. It cannot model a variable depth of streambed. The latest version (3.0) can accommodate multiple culverts. It will calculate culvert capacity for inlet control, pressurized, and outlet controlled embedded culverts. It directly calculates culvert hydraulics using headloss equations. The tailwater is modeled using a downstream channel cross-section. It easily generates tabular and graphic reports.

HY7 and **HY8** are companion hydraulic models developed by Federal Highways Administration. HY7 is an open channel flow model and HY8 calculates hydraulics of culverts. HY8 can analyze multiple parallel culverts of different dimensions and elevations. It can model a variety of culvert shapes but it will not model bed material in the culvert. It calculates inlet and outlet control and sub- and supercritical flow. It will calculate roadway embankment overtopping. HY8 version 6.1 provides tailwater options that include user-defined tailwater rating curves having up to 11 rating curve points; calculation of uniform flow in the downstream channel for regular or irregular channel cross-sections having up to 15 cross-section points and roughness assignments; or a constant tailwater elevation. Graphic and tabular reports include water surface elevation, discharge, velocity, rating curves, and more.

CulvertMaster™ (version 3.0) is a commercial backwater model by Haestad Methods. It can model multiple culverts with different dimensions and elevations and composite profiles can be used within a culvert. It includes modules for quick calculation of specific characteristics, culvert dimensions based on culvert size, skew and road fill, road overtopping, and tailwater curves or tailwater channel. It includes the option of a flat bed within the culvert but it will not model a variable roughness of the bed and culvert. Tailwater options include channel cross-section, variable cross-section roughness, and overbank flows assuming uniform flow in the downstream channel or a tailwater rating curve. Graphic and tabular reports include water surface elevation, discharge, velocity, rating curves, and more.

FlowMaster™ (version 6.0) is a companion to CulvertMaster by Haestad Methods. It is a program for the design and analysis of pipes, ditches, open channels, weirs, orifices, and inlets. It

is a one-dimensional model based on open channel and pressurized flow, not backwater calculations. FlowMaster's "Hydraulics Toolbox" can solve or rate any unknown variable using common hydraulic formulas. FlowMaster's inlet computations comply with the latest FHWA Hydraulic Circular Number 22 and AASHTO inlet computation guidelines.

Application of these models and proper interpretation of output requires that the user have a background in surface-water hydraulics. Even with sufficient experience, however, it is not always easy to determine what data are necessary to adequately define the physical system for numerical analysis. Similarly, determining whether or not the output from a model adequately represents the real-world situation can be very difficult.

The hydraulic programs listed above consider 1-dimensional flow, where the following general assumptions are applied: the slope of the channel bottom is small, the channel is prismatic and lateral inflows or outflows do not exist, head losses are determined considering uniform flow, and flow can only move in the downstream direction.

In more complex hydraulic problems, 2-dimensional modeling can be performed using finite element methods. For example, 2-dimensional analysis could be used to better model the effects from intersecting flows within a floodplain, tidal influences, and rapid variations in velocity within a tightly meandering river.

Generally speaking, computer programs that use finite element theory are very difficult to use and require a great amount of expertise in creating, calibrating, and interpreting models. For most cases, 1-dimensional analysis will be accepted in performing hydraulic modeling and design for fish passage projects.

3.4 Applied Hydraulics for Fish Passage Road Crossing Design

3.4.1 Flow Path Geometry for Embedded Culverts

When culverts are designed to be embedded, it is necessary to account for the reduced cross-sectional area when conducting open channel flow calculations such as Manning's equation. Figure 3-14 illustrates the defined elements d_b and d_w pertaining to the embedment depth and the water depth above the embedment, respectively. The equations in Table 3-5 can be used to calculate the embedment area, flow area, and other geometric properties of the embedded culvert.

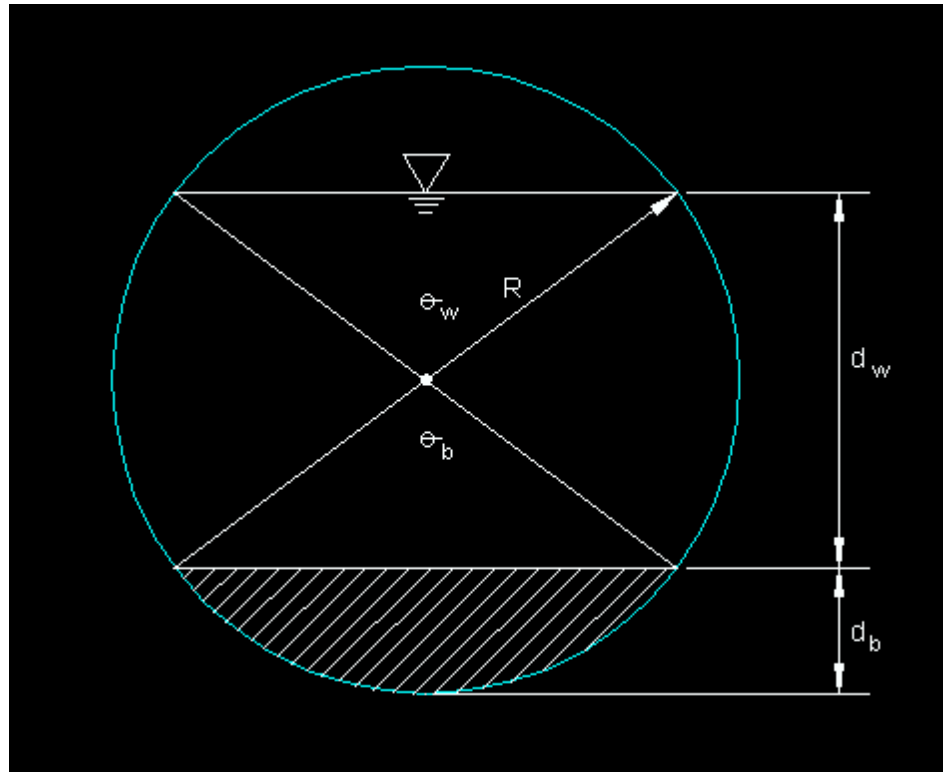


Figure 3-14. Elements of circular geometry used to calculate embedment area.

Table 3-5. Equations for calculating area and perimeter of embedment in circular pipes.

Depth of Embedment	d_b
Angle of Embedded Area	$\theta_b = 2\cos^{-1}[(R-d_b)/R]$
Embedded Area	$A_b = R^2 [\theta_b - \sin\theta_b] / 2$
Width of Embedment Surface	$P_b = 2R\sin(\theta_b/2)$
Embedded Pipe Perimeter	$P_p = 2R\cos^{-1}[(R-d_b)/R]$
Depth of Water	d_w
Angle of Water Surface	$\theta_w = 2\cos^{-1}[(R-(d_b+d_w))/R]$
Flow Area	$A_w = R^2 [\theta_w - \sin\theta_w] / 2 - A_b$
Width of Water Surface	$W_w = 2R\sin(\theta_w/2)$
Wetted Pipe Perimeter	$P_w = 2\theta_w - P_p$
Total Wetted Perimeter	$P = P_b + P_w$

The area of embedment in elliptical pipes can be approximated with the same equations, with pipe rise substituted for diameter. More exact results can be calculated with the following equation:

$$A = b (\text{pipe rise})^a \quad \text{Equation 3-7}$$

The coefficients a and b are given in Table 3-6. Note that two sets of coefficients are given, for corner radii of 457 mm (18 in) and 787 mm (31 in). These coefficients were developed by regression analysis from the exact tabulated areas (Maine DOT 2002).

Table 3-6. Function coefficients for open area in embedded elliptical pipe.

Corner Radius		Depth of Embedment			
		0 in	6 in	9 in	12 in
18 in	a	2.246	2.316	2.371	2.428
	b	0.743	0.613	0.530	0.453
31 in	a	2.260	2.291	2.320	2.351
	b	0.631	0.571	0.524	0.475

Note: Open Area (Ao, in ft²) = b x (Pipe Rise)^a, where Pipe Rise given in ft.

3.4.2 Composite Roughness

When there is more than one roughness condition present, it is possible to develop a single n-value based on a weighted average of the roughness value in relation to the wetted perimeter of its flow segment. This weighted roughness coefficient is also called a composite roughness coefficient. In equation form, it is

$$n = \frac{n_1 p_1 + n_2 p_2}{p_1 + p_2} \quad \text{Equation 3-8}$$

where,

- n = weighted roughness coefficient,
- n₁, n₂ = roughness coefficient of culvert and channel substrate, and
- p₁, p₂ = wetted perimeter of culvert walls and channel bottom.

It is recommended that weighted roughness coefficients be used when calculating flow conditions in culverts containing channel substrate, such as for bottomless culverts or embedded culverts. The length of the wetted perimeter of the substrate area can be calculated using methods presented in Section 3.4.1.

If a site has actual field data that provides paired data relating the river stage to a discharge Q, a single n-value may be back-calculated for each data pair using Manning's equation. In cases where a high level of accuracy is desired, this field-calibrated n value may be compared to the composite n-value calculated for individual channel sections. It is worth emphasizing, however, that the calculated n from field data for stage-discharge data pairings is valid only for that combination. It is not recommended that single n values as determined from field data be applied to other stage or discharge conditions. It is more reasonable to assign individual "n" values to localized and quantified subsections of the channel cross-section.

3.4.3 Weir Controls

Weir controls provide a convenient measure for controlling water depths upstream of the structure. Most often with fish passage road crossing structures, a weir will include a notch or other opening located at the deepest part of the channel, and an elevated crest that is highest at the stream banks and slopes downward towards the notch. These features ensure that, during low flow conditions, the flows are channeled through a relatively narrow area, rather than being spread out across the width of the channel.

Generally, weirs notches are constructed with simple geometric shapes to simplify flow

measurement. However, some of the flow measurement accuracy is lost as soon as the downstream water level submerges the crest of the weir, and a submerged crest is almost always the preferred condition with fish passage facilities.

Weir controls may be used to establish a tailwater control point below the culvert outlet. The standard contracted weir is one whose crest and sides are sufficiently far away from the channel sides to allow contraction to develop. A V-notch weir is especially useful when the lowest flow rates are small. The following equations are the weir formulas that relate discharge to the depth of flow over the crest:

- Standard contracted weir: $Q = 3.33 (L - 0.2H)^{3/2}$
- V-notch weir: $Q = C_d (\tan \theta/2) H^{5/2}$

3.4.4 Headwater

The constriction caused by the presence of the road crossing structure in the natural stream path may cause an increase in depth of flow in the channel upstream of the crossing. This total water depth, called the "headwater", represents the potential energy necessary to force the water into and through the culvert.

The calculation of headwater depth varies depending on whether the culvert flows under the influence of inlet control or outlet control. With inlet control, the culvert flow at the inlet is in the supercritical flow regime. With outlet control, the culvert is flowing in the subcritical flow regime. Subcritical flow is generally preferred for fish passage. Under the high flow conditions associated with peak flood events, it is feasible that even large culverts may become submerged at the inlet, increasing the chances of inlet control.

3.4.4.1 Inlet Control Headwater Calculation

If the culvert configuration is operating in supercritical flow, the headwater must be determined by reference to empirical relations which are based upon culvert model studies. The empirical relations correlate headwater to discharge and culvert face geometry. Since inlet control only occurs for supercritical flow, the parameters of barrel geometry, slope, roughness characteristics, and (usually) tailwater do not influence the headwater.

The empirical relations for various culvert shapes and materials are represented graphically by nomographs found in FHWA HDS #5. These relations are included in most culvert analysis software programs as well.

3.4.4.2 Outlet Control Headwater Calculation

If the culvert configuration is operating in subcritical flow, the headwater may be determined by analysis of the various losses which must be overcome by the potential energy represented by the headwater. These include the velocity head loss, entrance head loss, and friction head loss.

$$H = H_v + H_e + H_f \quad \text{Equation 4-10}$$

Calculations that determine headwater for culverts flowing under outlet control are included in most culvert analysis software programs. Several guidebooks for traditional culvert design can provide more information regarding the principles of outlet control flow

3.4.5 Outlet Velocity

Due to the decreased area of flow in the culvert, the flow velocity increases in the culvert from the natural stream velocity. The culvert outlet velocity, then, is an important parameter to be derived from the hydraulic analysis of the designed culvert. An excessive outlet velocity may be influential in the ultimate design configuration of the culvert. It usually is not cost effective to configure the culvert entirely upon the basis of outlet velocity.

3.4.5.1 Inlet Control Outlet Velocity

For inlet control configurations (supercritical flow), the backwater profile in the culvert tends toward normal depth as water passes from upstream to downstream. Since culverts are relatively short, normal depth of flow is rarely attained. Rather some depth greater than normal depth usually occurs at the outlet end of an inlet control culvert operation.

The calculation of the actual depth of flow at the outlet end of an inlet control culvert is tedious and usually unjustified. The conventional approach is to assume normal depth of flow at the outlet and use the associated cross-sectional area of the culvert barrel in the continuity equation to estimate culvert outlet velocity. This results in a conservative estimate of outlet velocity for manual calculations.

3.4.5.2 Outlet Control Outlet Velocity

The outlet velocity for a culvert operating in outlet control is calculated according to the following.

- Condition A-Outlet velocity is based upon full flow at the outlet of the culvert.
- Condition B-Outlet velocity is based upon the tailwater depth at the outlet of the culvert.
- Condition C-Outlet velocity is based upon the critical depth of flow in the culvert.

Most fish passage road crossings will be designed to flow under Condition B.

3.4.6 Estuary Flows

Fish passage designs for facilities located in estuaries must account for the unique hydraulic conditions caused by changes in tidal elevation. During ebb flow conditions when the tidal elevation is dropping, stream flow velocities are greater than the normal stream velocity due to the additional outflow of the tidal storage prism. Also, streams located in tide flats may exhibit water depths that are so shallow during low tide as to become impassable. HDM Index 821.5 specifies that for road crossings located where tailwater elevation is controlled by tides, special studies are normally required to determine the tailwater stage consistent with the design storm frequency of the facility.

Considering the difficulty in achieving the standard fish-passage criteria, new culverts that create a barrier due to tidal extremes are not generally permitted, and removal is a preferred action for restoration. Where removal is not possible but there is a need to achieve the best possible fish-passage restoration, objectives that are different from the standard fish-passage criteria might be acceptable. Defining alternative objectives should be done in conjunction with a careful and thorough review of allowable upstream water levels and timing.

3.5 Basic Culvert Design for Fish Passage

3.5.1 Culvert Alignment

Guidelines regarding culvert location and alignment presented in HDM Topic 823 generally recommend alignment of the thalweg of the stream with the centerline of the culvert. The HDM additionally recommends, however, that for economic reasons, small skews should be eliminated, moderate skews retained and large skews reduced.

Road crossings requiring fish passage are strongly encouraged to retain the natural alignment of the stream, regardless of the skew. Alignment of the culvert centerline with the channel approach angle should reduce the likelihood of the crossing plugging with debris during storm flows and minimize hydraulic turbulence which might impede fish passage. Additionally, curvature within culverts should be avoided at all fish passage road crossings.

As a general guideline, cases in which it is unavoidable to have a channel approach angle less than 30°, and cases where the channel curves or meanders at the road crossing location, may be better served by providing a bridge crossing instead of a culvert.

Where opportunities arise to evaluate alternative road crossing locations, preference should be given to sites where the stream channel has straight alignment 100 feet above and below the culvert. At a minimum, there should be at least 50 feet of straight alignment in the natural stream channel above the culvert.

3.5.2 Culvert Slope

The slope of most culverts installed without consideration to fish passage are placed so that the upstream and downstream flow line elevations approximately match the natural streambed elevations at those locations. HDM Topic 823 describes cases for non-fish passage culverts where, as a practical matter, the culvert flow line elevations may differ from the thalweg elevation of the stream. These cases include very high fills, where it may be uneconomical to install and operate a culvert at the thalweg elevation, and cases where the drop in stream elevation from one side of the roadway to the other is considerable. These latter cases might provide a broken back profile that incorporates a steep upstream segment of the culvert with a mild slope unit usually in the downstream end of the culvert. These common slope configurations for non-fish passage culverts are illustrated in Figure 3-15. As shown, there is little likelihood either of these configurations would obtain approval from fisheries agencies as acceptable for fish passage.

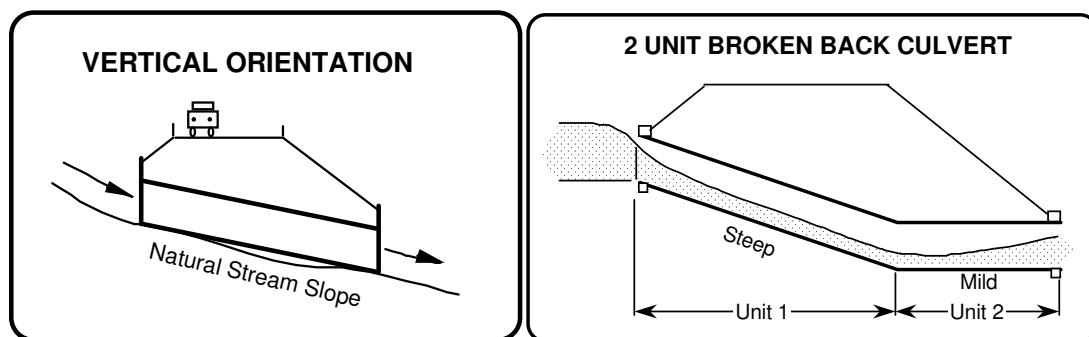


Figure 3-15. Common slope configurations for non-fish passage culverts.

Road crossings designed to accommodate fish passage generally place the bottom of the road crossing structure below the elevation of the stream thalweg (Figure 3-16).. In the case of bridges and culverts designed through the stream simulation or active channel options, the structures are generally placed to ensure that native stream bed material is recruited into and through the crossing to maintain the same gradient as the stream channel. Culverts designed by the hydraulic design option may vary the culvert slope as long as the velocities and water depths occurring within the fish passage flow range satisfy conditions related to the swimming capabilities of the target species. Specifics regarding design of the structure slope vary according to the culvert design option selected, and they are detailed in Chapters 4, 5 and 6.

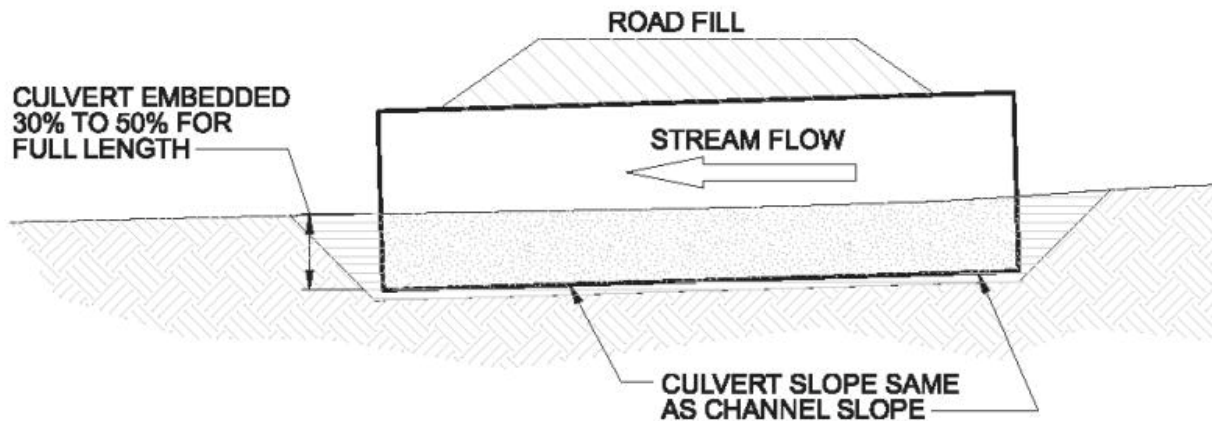


Figure 3-16. Preferred slope configurations for fish passage culverts.

3.5.3 Culvert Material and Shape

No single culvert material or shape is best for all fish passage road crossings. Figure 3-17 shows shapes of culverts commonly used for fish passage road crossings. All of these culvert shapes can be sized large enough to accommodate natural streambed materials within them. If the bankfull channel widths and bed characteristics are the same as the natural channel, and there is adequate hydraulic capacity to sustain the stream simulation characteristics though the life of the project, there is little difference among these designs from the point of view of fish passage performance.

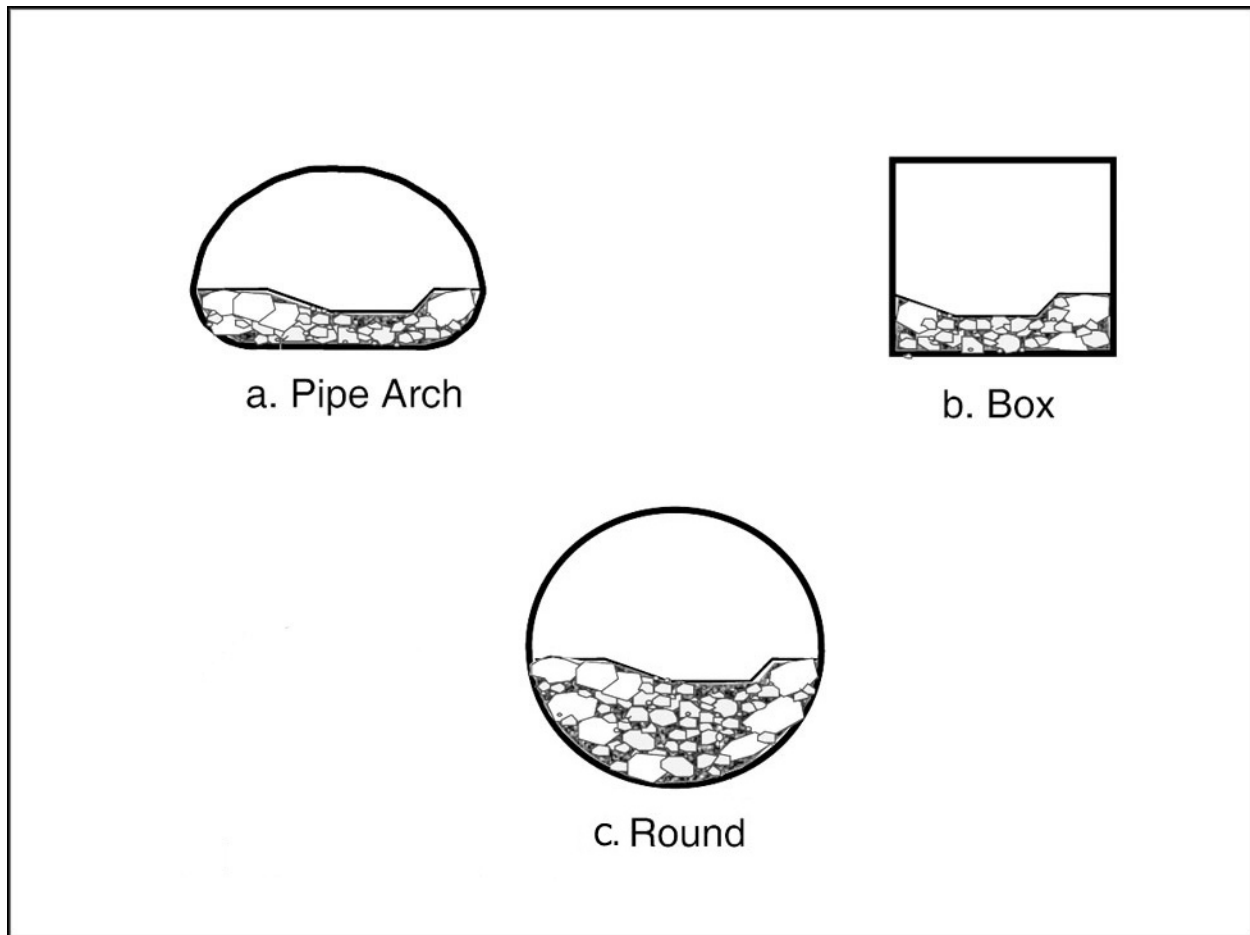


Figure 3-17. Common shapes for fish passage culverts, showing typical accommodation for embedment.

The shape and material of a culvert can substantially affect the initial and life-cycle costs of a project however. Cost differences may be due to material costs, project sequencing, limitations of project duration, competence of soils for structural loading, location and availability of materials, excavation volume, excavation limitations such as bedrock, and the cost of delivery to remote locations. Other differences include reliability, limitations of hauling over-sized materials, durability and resistance to corrosion and abrasion, risk of vertical instability, debris passage, hydraulic characteristics for stream simulation, and experience of the construction crew.

No universal recommendation is made for culvert shape or material. General categories that might be used to compare various culvert products for a project are:

- Full pipe;
- Precast or cast in place;
- Prefabricated or fabricated in place;
- Concrete, steel, or aluminum; and
- Arch, round, or box

Pipes are defined here as fully enclosed structures with an invert as an integral part of the culvert product.

An advantage of precast or prefabricated structures is that they can be installed in a single piece or a few pieces rather than being cast in place concrete or individual panels. In some cases structural plate pipes can be pre-assembled and installed in segments. The option of top loading a box culvert might be possible if a lid is installed after construction of the bed within the culvert.

Smooth-walled culverts have water velocities that are usually two to three times those in corrugated metal pipes when the slope, area, and other flow parameters are equal. Studies have shown that migrating fish, especially juvenile fish, use the corrugations along the pipe as resting areas as they migrate through the pipe. The passage enhancement is more pronounced with larger corrugations than smaller ones.

3.5.4 Culvert Size

There are several trade-offs of a larger culvert. A larger culvert obviously costs more, although the cost of culvert material for several feet greater diameter is often minor compared to the overall project cost. Sometimes there is greater cost in the extra excavation for installation, especially through a high road fill. There is also greater cost in materials and labor if selecting a larger culvert forces the design from a single piece corrugated pipe to a structural plate pipe. That threshold occurs at about twelve feet, which is generally the largest size of pipe available before going to a structural plate product. Some loading conditions may require structural plate pipes for smaller diameters.

There is also a lower practical limit to the diameter of culvert that can be constructed as stream simulation. A culvert with a diameter or rise of about five or six feet is a minimum because the bed cannot feasibly be constructed in a smaller pipe except with hand labor. Access may not be an issue if strong hand labor is available and the rock is not too large to manipulate by hand or with hand equipment. This size consideration should also be evaluated relative to future access for maintenance and repair.

3.5.5 Culvert Entrance Design

HDM Topic 826 provides guidance for the entrance design of non-fish passage culverts, noting inlet edge configuration is one of the prime factors that can influence hydraulic performance of culverts. However, these same entrance geometry refinements may be detrimental to conditions which promote fish passage.

The use of concrete aprons at culvert openings is not recommended for fish passage road crossings. The aprons have a smoother texture and therefore a lower roughness coefficient than the natural stream bed, which may increase velocities to the point of making fish passage difficult or impossible. Under lower flow conditions, aprons may produce sheet flow conditions that create a problem of shallow water depth. In cases where the use of an entrance apron cannot be avoided, the designer should depress the apron elevation below the culvert inlet, so that natural streambed materials can settle over smooth apron.

3.5.6 Culvert Outlet Design and Tailwater Control

Culverts designed for non-fish passage road crossings have outlet velocities that are usually higher than the maximum natural stream velocity. HDM Topic 827 provides guidance for determinations as to whether outlet velocities may be excessive and the design of corrective measures to reduce streambed scour and bank erosion below the culvert outlet. These measures, while appropriate for non-fish passage culverts, do not address the level of analysis or velocity reduction required to assure fish passage through a culvert.

Fish passage road crossings generally strive to keep flows subcritical and within 1.25 times the velocity of the natural stream. As a consequence, many of the standard concerns regarding excessive velocities at culvert outlets are not relevant to fish passage road crossing designs. Instead, outlet design for new fish passage road crossings focuses on the natural stream bed elevation at the outlet location of the culvert, along with the elevation of the tailwater control point located below the culvert.

Fish passage road crossings developed using the stream simulation or active channel design options are embedded culverts with inverts located below the natural stream grade. Analysis for determination of the culvert outlet elevation is the key element of outlet design for these two options. Generally, the outlet design entails:

- For both of these cases, assume that the bed elevation at the outlet will be equal to the bed elevation at the tailwater control point. This assumption is a conservative acknowledgement that natural sedimentation processes are likely to result in a filled bed in the channel above the tailwater control elevation.
- For stream simulation outlets: having established the design bed elevation, the invert elevation for full barrel culverts is located such that the bed elevation lies between 30% to 50% of the culvert height. For bottomless culverts, it is first necessary to determine the largest anticipated scour depth, and insure that the culvert footings or foundation are located below that depth.
- For active channel outlets: having established the design bed elevation, the invert elevation is located such that the bed elevation is between 20% to 40% of the culvert height.

For new culverts designed using the hydraulic design option, a more rigorous analysis is required to establish the outlet design. Hydraulic computations are completed to determine velocity and water depth at the outlet of the culvert at the high and low fish passage flow rates, to insure they are within the swimming capabilities for the target species. In some cases, the only mechanism for achieving the design criteria is to develop a new, higher tailwater control elevation below the culvert location as a means to backwater the culvert outlet.

Cases involving existing culverts being retrofit for fish passage enhancement also require hydraulic analysis of the outlet velocity and water depth conditions. In retrofit cases where there is a scour hole located below the culvert, it is common to use grade control measures to develop a new tailwater control point. In cases where these grade control measures still result in a hydraulic drop at the outlet, it is recommended that a jump pool be provided that is at least 2 feet deep, and the total drop be limited to 1 foot.

3.5.7 Culvert Lighting

As directed in the CDFG *Culvert Criteria For Fish Passage* document, interior illumination is required for both new and replacement culverts that are more than 150 feet in length. This illumination can be in the form of artificial lighting or natural light. Natural light can be conveyed through the use of solar tubes or even grated drainage inlets to identify a couple of alternatives. The spacing of supplemental lighting sources (artificial or natural) shall not exceed 75 feet.

3.6 Other Channel Considerations

It is important for the designer to have a firm understanding of the character of the stream

upstream and downstream of a culvert or bridge. Proper design or analysis of open channel flow must include the consideration of such things as the sediment load of the streamflow, debris which may be carried by the stormwater, and the various means and ramifications of bank protection. The designer must have:

- knowledge of the stream's natural conditions,
- estimates of sediment and water discharge,
- a means of predicting the type and magnitude of potential stream response to the proposed highway facility,
- a knowledge of geology, soil mechanics, hydrology, and hydraulics of alluvial streams. Alluvial streams have beds and banks composed of clay, silt, sand, or gravel and various combinations of these materials that have been transported by and deposited in water,
- and bed elevation change

3.6.1 Sediment

As previously mentioned in this chapter, the bed load in a stream is defined as sediment that moves by rolling, sliding, or skipping along the bed and is essentially in contact with the stream bed. Effectively, sediment load means the same thing. The bed load or the sediment load in stream flow may result in one or more of three things. Specifically, this material may cause:

- erosion (degradation of the channel),
- deposition (aggradation of the channel),
- or there may be a state of equilibrium in which any eroded material is replaced immediately by deposited material.

The bed load in a stream must be considered by the designer from the standpoint of its potential for damage to the stream, the roadway embankment, the drainage facility, or upstream or downstream property.

3.6.2 Debris

As runoff accumulates from a watershed, it naturally carries certain floatable material with it, which includes urban debris or trash. In watersheds with significant vegetation, because of the natural life and death processes of that vegetation and also because of cutting or clearing operations, debris or drift may accumulate at the surface of channels carrying the runoff. As this debris approaches the usually constricted highway drainage facility opening, it is deposited on the upstream side of the highway. Significant accumulations of debris can reduce hydraulic efficiency, cause local scour, and cause physical damage to the facility and adjacent property and features.

There are no comprehensive models for managing debris flow, debris plugging, or sediment failure probabilities. Regional models or experience may be available in some regions. Furniss et al. (1998) identified four primary mechanisms at 258 culvert failures during floods in 1995 and 1996 in Pacific Northwest forested watersheds. They identified debris flow, debris plugging, sediment slug, and hydraulic exceedance as primary mechanisms. Of these, debris and sediment combinations accounted for 91% of the failures. Of the failures due to debris plugging, 23% were initiated by debris smaller than the culvert diameter. These observations, on top of hydrologic modeling uncertainties, demonstrate uncertainties of predicting probabilities of culvert failures. Large tree-size wood is unlikely to pass through many structures.

Furniss et al. (1998) demonstrated that culverts with a width similar to the channel convey large debris more efficiently than culverts narrower than the channel and culverts with wide channels at the inlet. The widened channel at the culvert inlet allowed wood to rotate perpendicular to the culvert and plug the inlet.

Consideration for how material can be removed from the culvert may be an important design element. Management of the risk of debris by optimizing the culvert alignment was discussed in the section on alignment.

3.6.3 Bank Protection

Caltrans is in the process of piloting several projects which are combining RSP and vegetation - typically using RSP from the estimated scourable toe up to some elevation approximating "bankfull", with either vegetation, or vegetation over the top of buried RSP above this elevation. However, as yet there are not any specific design procedures or protocols and such designs are developed on a case-by-case basis - typically with HQ input.

Bank protection design for fish passage road crossing should refer to the California Bank and Shore document, while being cognizant of the desire of resource agencies to incorporate softer treatments where possible. The designer is advised to contact either their District Hydraulic Unit for input, or for unique or special situations to contact the Caltrans Bank and Shore Protection Committee (see HDM Index 802.3) for assistance. Additional information is presented in Chapter 8 as it relates to the use of rock weirs for grade control and channel stabilization.

3.7 Preliminary Design

The preliminary design process for a fish passage project is somewhat similar to the usual engineering procedures with the exception that more detail is required in this phase to assure that the structural and hydraulic features meet the specific passage criteria. This is necessary not only to assure subsequent design is carried out efficiently but also to confirm the passage design review, at this point in the process, by the permitting agencies involved is complete from their perspective. This stage of design is sometimes referred to as the "30% design level" and is a refinement of the internally developed and approved conceptual plan (approved by resource agencies).

Once the conceptual plan and supporting engineering information is confirmed within the agency, the development of preliminary engineering should progress to a level that reflect the required hydraulic conditions for water conveyance and fish passage through the defined times of passage and storm flow events.

Permitting agency review should be initiated at this point following the appropriate PDWT for general review and comments with the understanding that the final design approval will be on the final design documents and will include any appropriate permitting agency comments.

3.8 Final Design

Final design includes the process of refining the preliminary design as required from permitting agency review and the engineering detail developed from all the disciplines involved in the PDWT sequence. A "90 %" submittal is required in the final design process for internal review to assure compliance with the overall project intent both of engineering and environmental nature. The structures, hydraulics, soils and related engineering features shall then be completed in final design plan and specification format (100% submittal), as required for any CalTrans

project. This final design submittal shall address all internal and permitting agency concerns and be ready for advancing in the PDWT process toward District Engineer approval and advertising.

Elements of final design are likely to include:

- Bedding conditions and backfill.
- Installation conditions.
- Piping considerations.
- Debris control.